LOW DAMAGE DESIGN FOR SPLIT LEVEL CARPARK, HAZELDEAN BUSINESS PARK, CHRISTCHURCH

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ABSTRACT

Following the 2010-2011 Canterbury earthquakes, the concrete carpark building at the Hazeldean Business Park needed to be demolished, and the client was looking to replace the carpark with a steel framed building making use of a "damage control design" philosophy, while making the best use of the existing piled foundations.

This case study discusses our design process for the key aspects of the project, including the change in design from eccentrically braced frames (EBFs), with removable active links, to buckling restrained brace frames (BRBs), low-damage detailing of precast concrete lift and stair cores, complications around detailing ramps between split level, reuse of existing screw pile foundations, and considerations in the design of the ground slabs in relation to liquefaction risks.

The carpark was the second building designed by the author using Star Seismic® BRBs and this report will cover design procedures and learnings from both projects, as well as research following these projects.

Figure 1 – Photo from North-West Corner (source: T. Lamy Silva, Blueprint)

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Introduction

Following the Canterbury Earthquakes, the carpark at the Hazeldean Business Park needed to be replaced, and following on from the work Structure Design Ltd did on a new steel framed office building we were engaged to design a replacement carpark.

As a result of the Canterbury Earthquakes, the client felt there were negative perceptions towards reinforced concrete buildings and required the replacement carpark to be designed using structural steel. The client also had a number of buildings which required significant concrete repair, so were keen for "damage control philosophy" to be used.

Structure Design were engaged following concept design, where the intention was to replace the carpark like for like although lighter by substituting the concrete shear walls with EBFs, allowing the screw pile foundations to be reused.

This case study touches on the issues we had designing EBF's with removable active links, with emphasis on the reasons for changing to BRBs and discussion of the analysis and design process using BRBs.

During the design process the arrangement of the building was changed to incorporate office space on the ground level, which did not exist in the original building. The split level arrangement posed challenges when considering the diaphragms, which were tied together by ramps to limit the interstorey displacement effects on columns, stair and lift cores along the centre of the building.

The precast concrete stairs, stair core and lift core are designed and detailed with a low-damage approach. The ground slabs are not designed using a low-damage approach, where the costs of a low-damage design needed to be considered against the potential repair costs of a less resilient design.

There were difficulties involved with the review and reuse of the existing screw piles and this report provides some background to this process.

The design and construction of the carpark have proven very successful, with the client now looking to replicate the design approach on other projects they are involved with.

Description of Building

The Hazeldean Carpark is located at 14 Hazeldean Rd, Addington, Christchurch, it has a footprint of 2500m^2 with car parking over 6 levels, including a mixture of car parking, bicycle parking, office space, IT/comms and shower amenities on ground level (refer Figure 3).

The floor to floor height is 3.0m, 1.5m between the split levels, with a 72.8m longitudinal length and 34.8m transverse width (refer Figure 2). The 150mm thick composite floors have a fall of 175mm and are supported by 600mm deep welded beams spanning 17.2m.
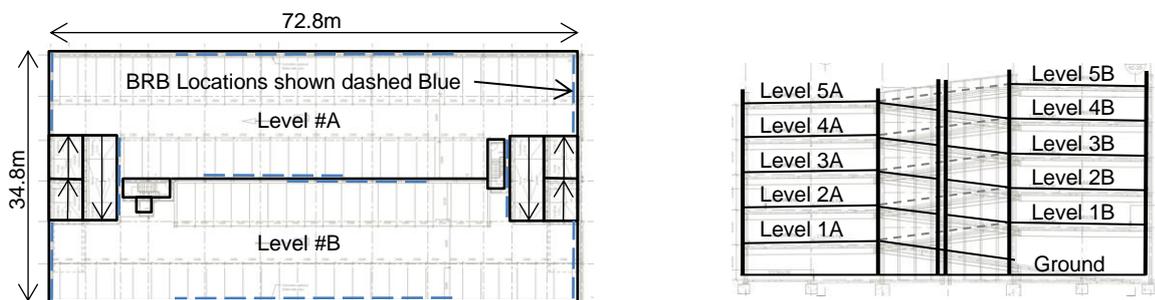

Figure 2.   Footprint and elevations of the carpark

The building is founded on screws piles reused from the original building, supplemented with additional piling at the corners of the building where the footprint of the new carpark has been extended and to support ramp locations (refer Figure 8).

The "up" ramps between split levels are designed to tie levels together, while the "down" ramps have been seismically separated to ensure the ramps do not act as a cross bracing element.

A 150mm thick ground floor slab has been used for parking and utility areas, while the office area has a 125mm thick slab founded on a 600mm deep hardfill raft, providing better protection against the potential effects on liquefaction, while balancing construction costs with potential repair costs.

The BRBs have been designed and manufactured by Star Seismic®, Utah, USA. The BRBs are site welded to the gusset plates, which were also designed by Star Seismic®. The BRBs are arranged concentrically with the columns.

The stair and lift core are 150mm thick precast concrete, which are designed to rock within the building, accommodate interstorey drift between the split levels, and have been detailed with crack inducers, at landing levels, to limit damage. The precast concrete stairs have been detailed to slide over landings.
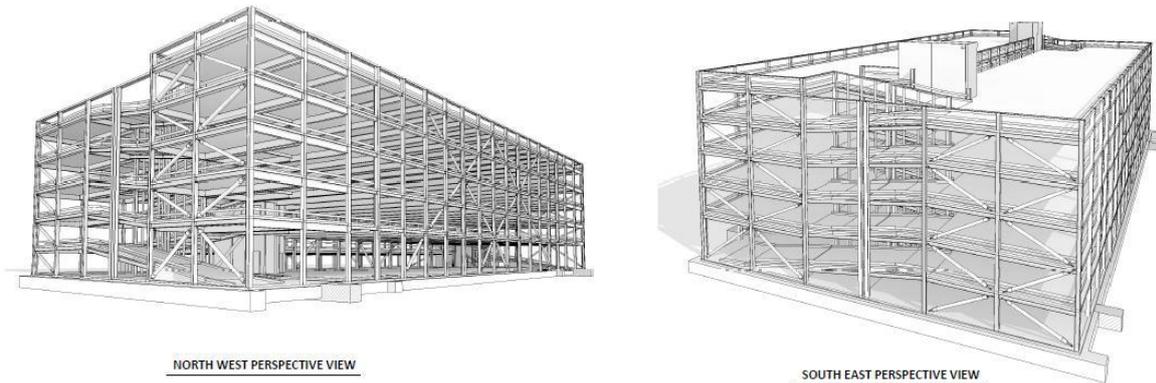

NORTH WEST PERSPECTIVE VIEW

SOUTH EAST PERSPECTIVE VIEW

Figure 3. Isometric views of the carpark.

Evolution of Design

Structure Design Ltd became involved with the project following concept design where the original engineer had proposed the use of EBFs with removable active links at shear wall locations. The idea was that if the original arrangement of the carpark was replicated with a lighter structure the existing foundations could be reused.

The new carpark was to have a minimum of 497 carparks, support the IT/Comms for the business park, have space for 190 bicycles, and would introduce office space on ground level, which hadn't existed in the original carpark. During preliminary design, we proposed changing the layout of the building to split-level, and increasing the footprint at the corners which better accommodated the office space, allowed a more efficient parking layout, as well as having a single traffic flow direction (refer Figure 4).
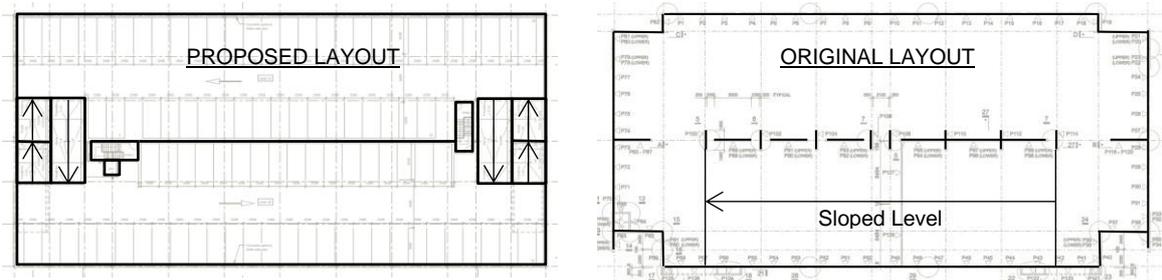

Figure 4. Comparison of new and old layouts.

When we came to designing the EBFs with removable active links, the industry recommendations (SESOC 2012) suggest that a ductility of 3.0 be used, this resulted in us needing a significant number of EBFs, and due to the overstrength requirements around the active link the associated columns, braces and beams were becoming large. The use of EBFs was not able to suit the new arrangement of the building without compromising the parking spaces, requiring many new screw piles and being fabricated from welded sections, the overall result was a significant budget blowout.

Structure Design had recently been collaborating with Beca on a building at the University of Auckland, where Beca had proposed BRBs (Beasley and Built 2012), we felt that BRBs could be efficiently used on the replacement carpark and proposed to investigate their use. The findings of our investigation and preliminary analysis, strongly favoured BRBs over EBFs, resulting in the client accepting our recommendation to use the BRB system.

The decision to use Star Seismic® over other suppliers was based on our experience with University of Auckland project, where they had been the approachable and helpful during early investigations. Liaising with the BRB supplier early helped speed up the overall design and procurement processes.

Structural Simulation

The building was simulated using ETABS 9.7.3 (unfortunately this was prior to the release of ETABS 2013, which now has the ability of simulate multiple towers) and the modelling of the split levels was complicated because the diagonal bracing elements ended up being split in two across the floor levels.

The decision was to create ETABS storeys at the A levels only (Refer Figure 2), having storeys at all levels would have resulting in all bracing and column elements being split in two (refer Figure 5), further complicating the processing of the outputs, which was done in Microsoft Excel.
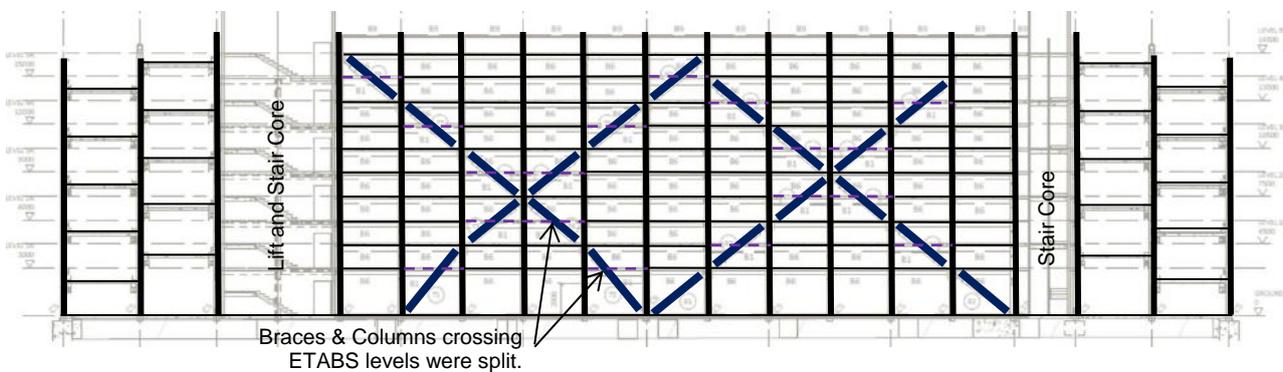

Figure 5.   BRB arrangement to centreline of building.

Two rigid diaphragms were defined and applied to the A & B levels respectively.

ETABS has a BRB database built in and these were used in the simulation, with an initial area modifier of 1.5 as recommended by Star Seismic® (Star Seismic 2012). The area modifier is essentially a reflection of the stiffness of the brace and this factor is altered, along with the brace size, in consultation with Star Seismic based on the demand on the brace.

All braces and beams had moment end releases. The exception was for collector beams, where the BRB connected to a column. This was to reflect the fact that the gusset plates would provide some rigidity at these connections. The base of columns were typically pinned, except where they are cast into the foundation at BRB locations.

Following the initial run of ETABS, P-delta checks were carried out and it was found that P-delta actions needed to be considered. These forces were calculated in a spreadsheet and applied as a user defined quake load in the load cases.

Screw piles were modelled in ETABS with springs at their base, the value of which was supplied by the piling contractor, a lateral spring stiffness was also applied to the foundation beams and piles. Pile groups were modelled as a single pile with the vertical spring stiffness factored up by the number of piles in the group.

The decision on what ductility to use for the building was made with reference to the US loading standard, which suggests a Response Modification Coefficient, $R = 8$ and Deflection Amplification Factor, $C_d = 5$ (ASCE/SEI 7-10).

These factors do not correlate directly to New Zealand Standards for determining seismic actions. By investigation the derivation of the seismic coefficients in the US standards, we determined that the R factor correlated to k_μ/S_p. We have since found that the relationship is more accurately represented as:

$R = R_\mu \times R_s$ (Uang & Maarouf 1994)   (1)

Where the formula to determine the reduction factor for ductility, R_μ, (Mahmoudi & Zaree 2013) is the same as the formula for k_μ (NZS1170:2004) and the basis for the reduction factor for overstrength, R_s, (Mohammadi & Naggar, 2004) is similar to the basis of $1/S_p$ (NZS1170:2004).

$$\text{For } T_1 > 0.7\text{sec } \mu = k_\mu \tag{2}$$

$$\text{Therefore, } \mu = R \times S_p \tag{3}$$

Based on the above formula, the response Modification Factor, R = 8, correlates to a structural ductility of μ = 5.6. We settled on a ductility of 4.0, which is perhaps not considered an ideal "low-damage" level of ductility for conventional structures, but seemed to be a good compromise considering the desire to reuse the existing foundations.

The Deflection Amplification Factor, C_d, influences the design in a couple of ways, the first relating to the assessment of building displacement and the second in consideration of overstrength actions induced in collector beams and columns from the displacements. As we had selected a ductility relating to an R value lower than 8, we were also keen to use a reduced C_d to reflect this. Research suggests that the relationship between C_d & R is not linear, so care must be exercised when selecting an appropriate C_d factor for cases when R ≠ 8 (Uang & Maarouf 1994 and Mahmoudi & Zaree 2013).

For building displacements, we chose to use C_d = 4.0, equivalent to the structural ductility we had chosen and slightly higher than scaling down from the C_d recommended in the US standard. When designing the collector beams and columns, the bending actions from an elastic analysis (μ = 1.0, S_p = 1.0) were combined with the overstrength axial forces, effectively taking C_d = R.

The C_s factor (NZS3404:1997), used in the design on conventional frames, is not applicable to BRB systems because they have similar capacity in tension and compression, and there is no need to consider second order effects from brace elongation and buckling. For similar reasons, where it was not practical, we also chose not comply with the NZ requirement to having opposing pairs of concentric braces.

Design using BRB Frames

Star Seismic[®] have a library of BRB sections built into ETABS and the analysis, although very iterative, is fairly straight forward. In the US, BRB manufacturers have a unified approach to the design process when using BRB's, which is well illustrated in a design flowchart (Robinson, Kerstin & Saxey 2012).

The process can be quite involved early in the analysis, when you have estimated BRB sizes and start with an area modification factor of 1.5. At this stage of the design it was important to have spreadsheets set up to automate the processes of updating P-Delta effects, response spectrum scaling and sizing of BRBs.

Also, with having to make iterative changes to the ETABS simulation it is important to have the spreadsheets set up to also double check ETABS input for each run.

To calculate the area modification factor, Star Seismic[®] were provided with workpoint to workpoint set out and the estimated sizes of the collector beams and columns, as well as the size and load on the BRB's themselves. They use the setout parameter to calculate the true length of the BRB, considering the size of gusset plates and then use the loads to calculate the area modification factor. As the length of the brace directly influences that area modification factor, it is also important that any other setout constraints are communicated, such as the depth of slab, which may affect the length of the brace.

As the design progressed, the model became less sensitive to the changes in area modifications and eventually the BRB sizes did not need to change in the model. Once we had convergence in ETABS, the final loads and set out were provided to Star Seismic[®], who confirmed back the overstrength forces, for design of the collector beam and columns.

When the collector beam and columns have been sized, ETABS is updated and rerun, it is possible a couple more iterations may be required to complete the analysis.

When the BRB order is placed with Star Seismic[®], they design the gusset plates and associated welds, they also provide full calculations for the design of the BRBs which will be based on the actual properties of the piece of steel they intend using for each brace. At this point the area modifications may change once again, although for the projects we've been involved with the analysis results were not affected.

Star Seismic® also provide shop drawings covering the BRB set out, detailing and tables summarising BRB sizes, welds and gussets. This information was to be provided after building consent documentation, and we chose to provide placeholder drawings in our documentation which had replica tables providing sufficient information for pricing, and we clearly noted across the drawings that the information was subject to change by Star Seismic® and to refer to their shop drawings. In the earlier building we did, we didn't have the clear note across our drawings and when the Star Seismic® shop drawings were issued this lead to confusion on site.

Collector Beams, Columns and Gussets

The design of collector beams and columns is relatively straight-forward, using the overstrengths provided by Star Seismic®, and following the standard design procedures for conventionally braced frames (Feeney, Clifton, 1995). We utilized the overstrengths Star Seismic® provided as their calculations were based on the actual steel properties, strains, testing, and using procedures which are tailored to BRB design. We were uncomfortable using the overstrength factor provided in the NZ steel standard (NZS3404:1997) as these factors don't cover the grade of material typically used for BRBs.

As gussets have an influence the BRB set out and design, Star Seismic® provided the gusset plate design and detailing, and the gussets were supplied by the NZ structural steel contractor.

Collector beams were modelled and detailed with moment fixity at the ends which intersected with BRB connections. The design actions were a combination of the overstrength axial loads and the moments from the elastic analysis, to account for moments induced by inelastic deformation in the BRBs.
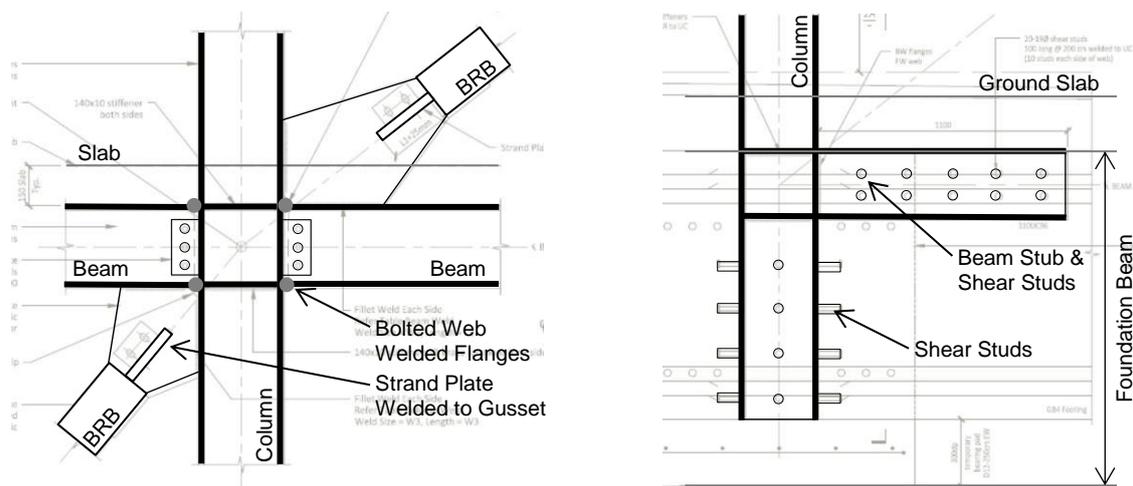

Figure 6.   BRB Connections at Upper Floors and at Base.

To help spread loads on to the foundation, the columns around the building were closely spaced, which meant that there were a number of locations where there was insufficient gravity load to counter the uplift overstrengths, resulting in column base details which needed to be cast into the foundation beams (Figure 6) This arrangement resulted in columns with moment fixity at the base, and similar to the collector beams the axial overstrengths were combined with the elastic moments from ETABS.

Floor Diaphragm Design

The 150mm thick composite steel/concrete decks, with trapezoidal profile, are designed to support the gravity loads and as a diaphragm, holding the building together during a seismic event. The decks are support by 600mm deep welded beams, which have been designed as composite using COBENZ, end connections of these beams were simple web plate configurations, the tolerance in the bolting was sufficient to accommodate the building drifts.

The floor plate in each split level is a relatively simple diaphragm, where the strut and tie model was essentially a beam analogue (Sabelli, et al., 2011). The elastic design actions were calculated based on the methods recommended in the US (Moehle, et al., 2010), which we also confirmed were in excess of the overstrength forces along each line of bracing. To add a further assurance and to simplify construction, we kept the detailing of the floors at all levels the same as the worst case.

The welded beams were considered to act as collectors in the diaphragm with the shear flow in the shear studs designed for a combination of gravity and 1.5 times the μ = 1.0 seismic actions (Sabelli, et al., 2011 and SESOC, 2012).

Particular attention was required for the "up" ramps which tied A & B levels, which have been designed based on the axial capacity of the steel support beams, with seismic separation to the ramp topping slab. The "down" ramps are split mid length to seismically separate the diaphragms. We originally attempted to design the ramp trimmer without a seismic separation, and we found ourselves chasing our tails, as the section sizes got larger they acted as a stiffer brace drawing further bracing loads and it was impossible to design them to remain elastic.
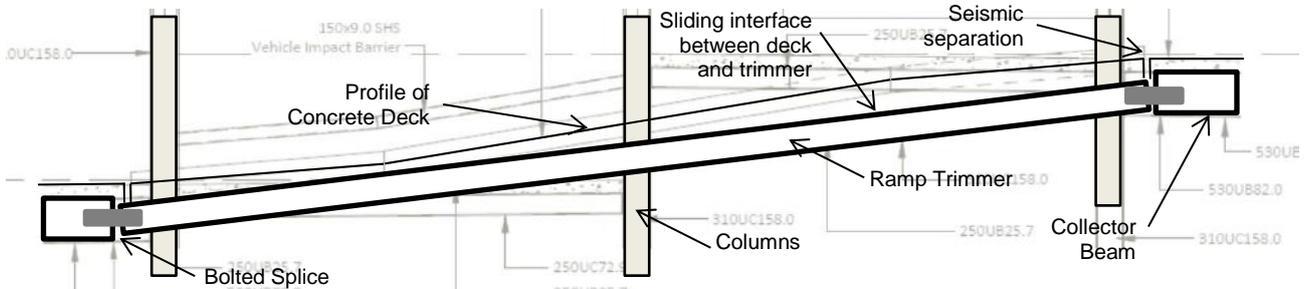

Figure 7.   Ramp deck profile shown against ramp trimmer

The ramp trimmers have bolted connections to the collector beams in the floor diaphragms. The bolted connections are designed for 1.5 times elastic actions and considering strength hierarchy. The plates, which are permitted to yield, are designed for 1.5 times elastic actions, while the welds and bolts, which have no ductility, have been design with further capacity to ensure any yielding is confined to the plates.

The concrete ramp follows the slope profile required for a trafficable ramp (refer Figure 7), and is designed to slide over the structural steel trimmer beams which are straight to limit the potential for buckling. The concrete deck, which spans between the steel trimmers, has 5mm separation to the main floors to limit the shear demand on the ramp.

Lift and Stair Core

The lift and stair cores are design to rock with the building, requiring careful detailing of the 150mm thick precast panels. Precast concrete stair flights are detailed to slide over landings, which are also partially precast.

The precast wall panels are cast with debonded steel plates extending out the bottom of the panels at ground level, these sit into RHS voids cast into the ground slab and are then grouted. Each of these shear keys is designed to take the design shear load, even when it has partially withdrawn out of the floor when the wall panel rocks.

Wall panels were cast three storeys tall, spliced with Reidbar grout sleeves, and detailed at each level with sheet metal crack inducers cast into the panels, with debonding around the rebar extending 200mm below the crack inducer. Wall panels are bolted to eachother and the surrounding floors using angles with oversized holes to allow sliding between interfaces.

The landings of the stairs are partially precast, bearing on steel shelf angles on the sides and across the end, a topping is laid insitu to tie the "down" flight of stairs into the landing along with the side panels, the end panel is not tied to the landing, allowing it to rock independently of the landing and side panels. The landings are also placed with a void each side of the landing to limit the potential for ratcheting between the landing and the panel.

The design capacity of the panels was based on the induced displacements relating to the various modal shapes, and allowing for the different displacements of the split levels. This was undertaken by extracting the modal displacements from ETABS, with some manipulation, later version of ETABS are now able to output the displacements directly. The moments induced in the walls from these displacements, were calculated based on the cracked section properties of the panels.

Ground Slabs

The ground slabs have been cast on grade in the ultility and parking areas, while the office areas have been cast over a 600mm thick compacted hardfill raft, with geotextile fabric. We could have fully piled the floors and suspended the slabs, but this would have added significant costs, while not guaranteeing a low-damage solution.

In collaboration with the client, the decision was made to use conventional slabs on grade in the utility areas, where protection of the slab was not critical and could be repaired relatively easily, while the compacted hardfill raft was used in the office areas to provide some protection.

The intention of the raft is to limit differential displacement across the slab, if soil settlement occurs as a result of liquefaction. The raft also provides a zone for water pressures to dissipate into should sand boils develop on the site.

Screw Piling

The original piling contractor was engaged to investigate the integrity of the existing screw piles, confirm the design capacity of the existing piles, design and install additional piles and issue a new PS1 relating to all existing and new piling.

Damage surveys of the original building, taken following the Canterbury earthquakes, suggested very minor cracking had occurred in a couple of wall panels, which had been designed for ductility, and there was no evidence suggesting the building had shifted laterally any distance in relation to the piling. From this evidence, it was clear to us that the screw piles had not sustained anything near their design loading, and the expectation was that the existing screw piles would be in good condition for reuse.

Piles were withdrawn from locations that we expected had been highly loaded during the Canterbury earthquakes, but were not required for the new building. The splice collars, near the tops of the piles, are particularly prone to damage as they are typically only fillet welded joints on the shafts, so have little capacity to sustain lateral movements, the tops of several more piles were excavated to confirm that collars had not been damaged.

The withdrawn piles had their helix checked for damage, materials testing was carried out on the shafts to confirm there had been no micro-cracking. All piles passed inspection and pile capacities were recalculated based on the new strength reduction factors, which changed following the Canterbury Earthquakes, effectively reducing the design capacities of the original piles.

The design of the superstructure was completed in collaboration with the screw piling contractor, and careful modelling was undertaken, using compression only piles, to check existing footings and detail new foundation beams.

Generally, the existing piles were cut short, had new starter bars welded to the shaft, paint protected for durability, with foundation beams were cast over. With the exception of new tension plies, all starter bars into the foundation beams are straight bars extending 200mm into the beams, essentially creating a pinned connection which was unable to develop tension in the piles.
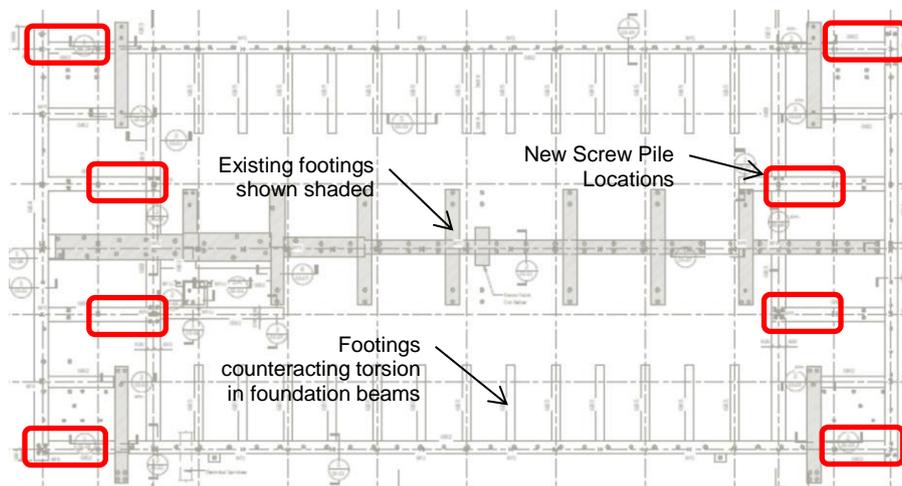

Figure 8.   Foundation beams and piling.

The footprint of the new building was slightly larger than the original building which resulted in some minor eccentricities, these effects were counteracted using footings laid perpendicular to the footings over piles.

Screw piles perform well for axial loads, but cannot handle excessive lateral displacements. Our approach was to rely on the passive soil pressures against the footings to limit lateral displacement under serviceability loads, while we have accepted that nothing can be done for an ultimate event. We have instead stated the maximum lateral displacement the piles can sustain while carrying the design actions, if the building does shift over 100mm following a major seismic event, rigorous investigation will be required to reconfirm the integrity of the piles.

When it came to issuing their PS1 for existing piling, the original contractor got stuck on needing to understand the actual loads that the building had sustained. We were in no position to provide such information, the best we could do was to infer, from the available information and from testing the piles, that the piles had not sustained loads anywhere near their design capacity. This was not good enough for the original contractor, which forced us to approach another contractor, who in coordination with their own structural and geotechnical engineers were able to justify issuing a PS1.

Summary

In summary, BRB frames have superior performance under seismic loads, particularly when compared with EBF's with removable links, and can provide the opportunity for replacement buildings to be built over existing foundations.

A "low-damage design philosophy" requires all aspects of the building elements and their interactions to be well considered and clearly detailed.

Acknowledgements

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